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QUINNIPIAC RIVER BASIN
HAMDEN, CONNECTICUT

LAKE WHITNEY DAM
CT 00119

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

AUGUST 1978

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SUPPLEMENTARY NOTES

Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,
Quinnipiac River Basin
Hamden, Conn.
Lake Whitney Dam

ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam is conprised of an earthen embankment on the upstream side of a near-vertical rubbles stone masonry wall, which is approx. 340 ft. in length and rises approx. 37 ft. above the original streambed elevation of the Mill River. The upstream earthen embankment is approx. 20 ft. wide at the dam crest and has an upstream slope with a maximum inclination of 3 horizontal to 1 vertical. The right portion of the spillway is a concrete compounded circular crest with a sloped downstream face. The left portion of the spillway is a side channel concrete ogee section.

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QUINNIPIAC RIVER BASIN
HAMDEN, CONNECTICUT

LAKE WHITNEY DAM
CT 00119

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

AUGUST 1978

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam:	LAKE WHITNEY
Inventory Number:	CT 00 119
State Located:	CONNECTICUT
County Located	NEW HAVEN
Town Located:	HAMDEN
Stream:	MILL RIVER
Owner:	NEW HAVEN WATER COMPANY
Date of Inspection:	MAY 30, 1978
Inspection Team:	DEAN THOMASSON
	HECTOR MORENO
	GONZALO CASTRO
	MIKE HORTON

The dam is comprised of an earthen embankment on the upstream side of a near-vertical rubble stone masonry wall, which is approximately 340 feet in length and rises approximately 37 feet above the original streambed elevation of the Mill River. The upstream earthen embankment is approximately 20 feet wide at the dam crest and has an upstream slope with a maximum inclination of 3 horizontal to 1 vertical. The right portion of the spillway is a concrete compounded circular crest with a sloped downstream face. The left portion of the spillway is a side channel concrete ogee section. The area below the dam is developed with industrial buildings, a high school, and residential structures.

Based upon the visual inspection at the site and past performance of the dam, the dam is judged to be in good condition. No evidence was observed of structural instability in the earthen embankment or the masonry wall. The condition of both the embankment and wall appears to be good. There are some areas requiring attention.

Based upon the size (Intermediate) and hazard (High) classification of the dam in accordance with Corps guidelines, the test flood will be equivalent to the

Probable Maximum Flood (PMF). Based upon our hydraulics computations, the spillway capacity is 9700 cubic feet per second, which is equivalent to approximately 21 percent of the Test Flood. Peak inflow to the reservoir is 48,600 cubic feet per second; peak outflow (Test Flood) is 46,500 cubic feet per second with the dam overtopped 4.4 feet. The peak failure outflow from the dam breaching would be 44,800 cubic feet per second.

An overtopping of the dam of 4.4 feet without breaching, would cause flooding and damage downstream with potential for loss of life. A breach of the dam would develop an 18 foot wave downstream of the dam, causing severe loss of life and damage to property.

It is recommended that further studies be undertaken to perform a more refined hydraulic/hydrologic study to determine the best way to increase the ability of the spillway to pass a greater percentage of the test flood.

An operation and maintenance plan should be instituted. The arch culvert outlet structure in areas of observed surface subsidence should be examined for possible partial collapse. The upper 3.5 feet (approximately) of the upstream face constituted by the sloping earth crest, should be protected from erosion.

The above recommendations and remedial measures should be instituted within one year of the owner's receipt of this Phase I Inspection Report.

Peter M. Heynen, P.E.
Project Manager
Cahn Engineers, Inc.

William O. Doll, P.E.
Chief Engineer
Cahn Engineers, Inc.

This Phase I Inspection Report on Lake Whitney Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

CHARLES G. TIERSCH, Chairman
Chief, Foundation and Materials Branch
Engineering Division

FRED J. RAVENS, Jr., Member
Chief, Design Branch
Engineering Division

SAUL C. COOPER, Member
Chief, Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionarly in nature. It would be incorrect to assume that the present condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions there of. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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Albert B. Hill, May 7, 1914

"Plans for Improvements to Buttress B-36
and Spillway Channel", Lake Whitney
Dam", New Haven Water Company
Hamden, Conn., August 1916

"Plan for Raising and Lengthening B-37, 38
Spillway, Lake Whitney Dam" (2) New Haven
Water Company, Hamden, Conn.
August 1916 and January 1917

"Plan of Powerhouse, Lake Whitney Dam" B-39
New Haven Water Company, New Haven, Conn.
Blair and Marchant, Inc., December 9, 1932

A set of "As-Built" plans for Lake Whitney B-40 to 45
Dam, Additional Outlet Facilities; Six
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"Location Plan and General Plan"
"Plan and Details"
"Section and Details"
"Structural Details"
"Plan and Details"
"Plans, Section and Details"

Malcolm Pirnie Engineers
Latest revision, March 1966

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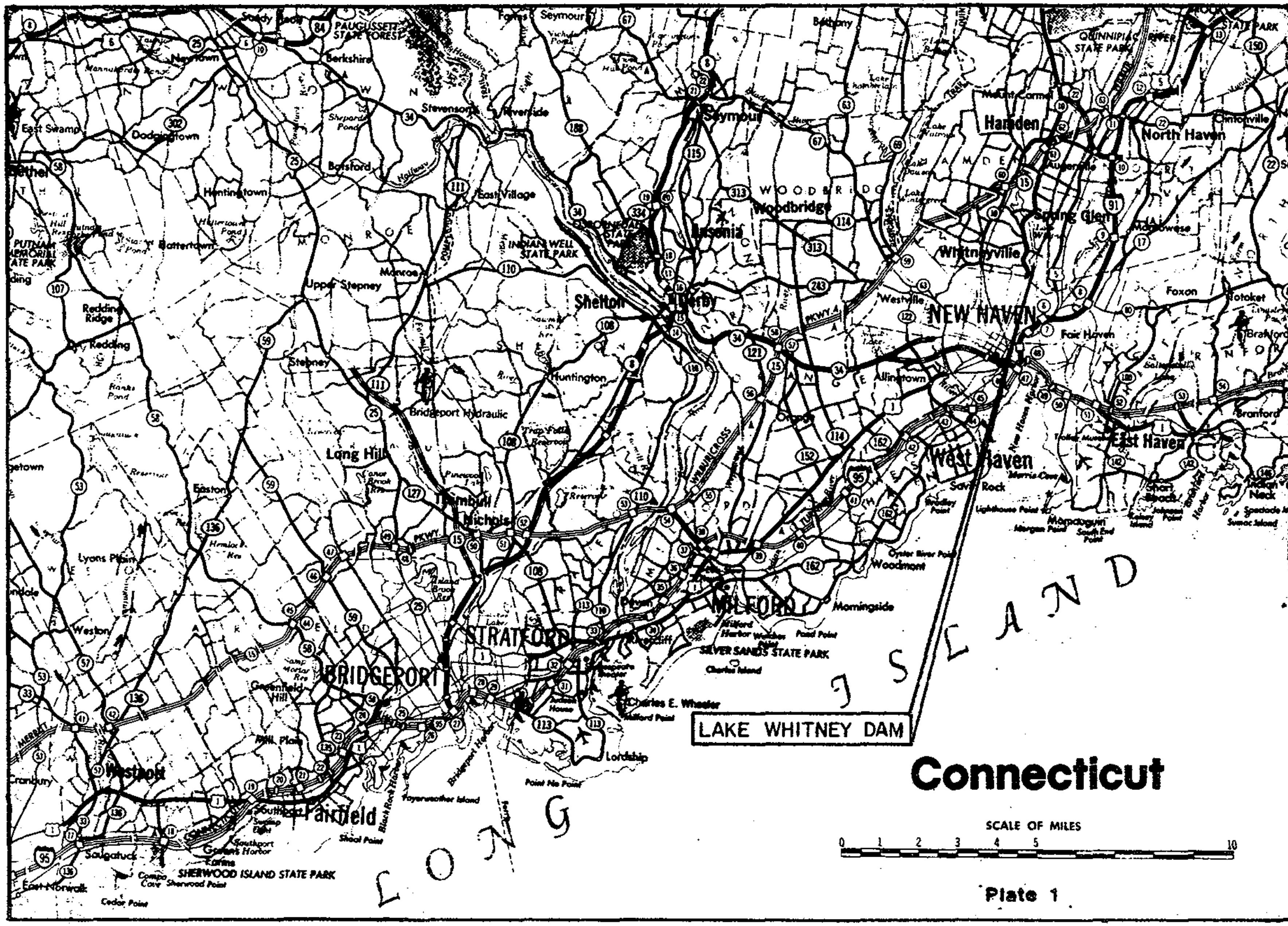
*See Special Note Appendix Section B Availability of Data.

OVERVIEW PHOTO

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS. CAHN ENGINEERS. INC. WALLINGFORD, CONN. ARCHITECT —— ENGINEER	NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS	LAKE WHITNEY DAM MILL RIVER	HAMDEN CONNECTICUT	DATE 5/30/78 CE # 27 531 GF PAGE x


Connecticut
SCALE OF MILES
0 1 2 3 4 5 10
Plate 1
LAKE WHITNEY DAM
LONG ISLAND
NEW HAVEN
BRIDGEPORT
STRATFORD
MILFORD
West Haven
East Haven
North Haven
Hamden
Seymour
Ansonia
Woodbridge
Shelton
Derby
Fairfield
Westport
Trumbull
Monroe
Newtown
Branford
Orange
SHERWOOD ISLAND STATE PARK
SILVER SANDS STATE PARK
INDIAN WELL STATE PARK
PAUGUSSETT STATE FOREST
Charles E. Wheeler
Lordship
Whitneyville
Westville
Bridgeport Hydraulic

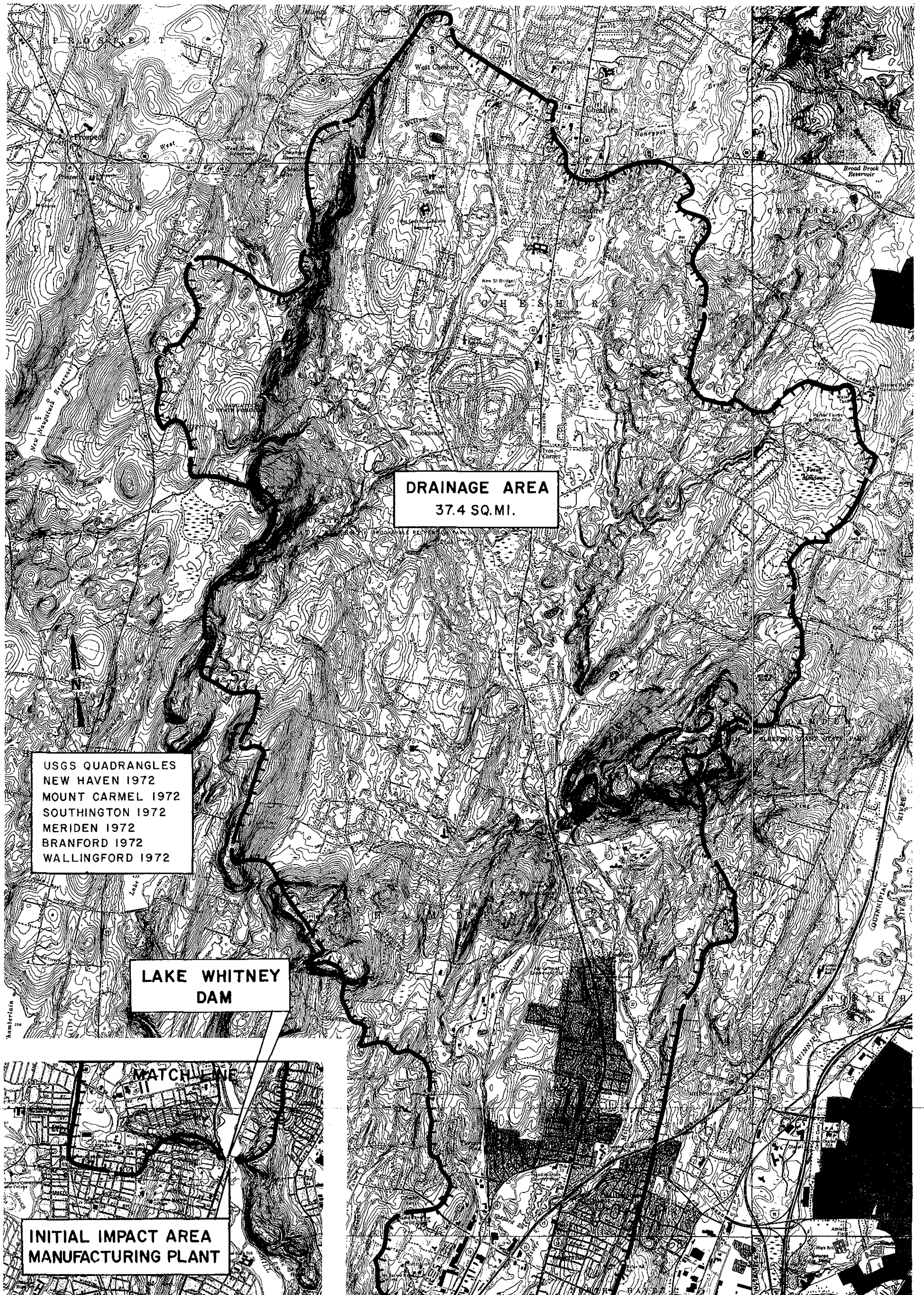

DRAINAGE AREA
37.4 SQ.MI.
USGS QUADRANGLES
NEW HAVEN 1972
MOUNT CARMEL 1972
SOUTHINGTON 1972
MERIDEN 1972
BRANFORD 1972
WALLINGFORD 1972
LAKE WHITNEY DAM
MATCH LINE
INITIAL IMPACT AREA
MANUFACTURING PLANT
MATCH LINE
CAHN ENGINEERS INC.
WALLINGFORD, CONNECTICUT
ARCHITECT-ENGINEER
U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORP OF ENGINEERS
WALTHAM, MASS.
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
LAKE WHITNEY DAM
MILL RIVER
HAMDEN, CONNECTICUT
Dwn. By J.P.
Ckd. By D.T.
App. By P.H.
SCALE: 1" = 4000'
DATE: 5/30/78

PLATE NO. 2

PHASE I INSPECTION REPORT

LAKE WHITNEY DAM

SECTION I

PROJECT INFORMATION

1.1 General

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the southwestern portion of the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of April 26, 1978 from Ralph T. Garver, Colonel, Corps of Engineers. Contract No. DACW33-78-C-0310 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

(1) Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

(2) Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dams.

(3) To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

(1) Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

(2) A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

(3) Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

(4) An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features on the dam which need corrective action and/or further study.

1.2 Description of Project

a. Description of Dam and Appurtenances - The dam is approximately 340 feet in length and the top is approximately 37 feet above the streambed elevation of the Mill River. The structure is an earthen embankment on the upstream side with an adjacent rubble stone masonry wall on the downstream side. The upstream face of the masonry has an 8 inch thick cover of concrete extending from the foundation in rock up to the maximum elevation of the original construction. From the downstream face to 2 feet horizontally into the masonry, the stone was laid in cement mortar. The remaining stone from the limit of the mortar to the upstream face of the masonry adjacent to the earthen embankment was dry laid. The original spillway was constructed in a manner similar to that of the dam. Later raisings included capping the spillway with concrete, and the construction of a concrete side channel spillway, both of which are located at the left end of the dam. As the dam presently exists, the right portion of the spillway is a concrete compounded circular crest with sloped upstream and downstream faces. The left portion of the spillway is a side-channel concrete ogee section with a vertical upstream face and a sloped downstream face. The blowoff and supply intake structures and the gate chambers are adjacent to the upstream and downstream faces of the dam, respectively. A 42 inch and a 24 inch steel pipe connects the intake structure and gate chamber at the right end and at the center of the dam, respectively.

The dam is located upstream of industrial buildings, a high school and residential/urban developments in the New Haven area.

b. Location - The dam is located on Mill River in a residential area in the Town of Hamden, County of New Haven, State of Connecticut. The dam is shown on the New Haven U.S.G.S. Quadrangle Map having coordinates of longitude W72° 54' 40" and latitude of N41° 20' 12".

c. Size Classification - INTERMEDIATE - The dam has 3600 acre feet of storage with the water level at the top of the dam, elevation 41.3, which is approximately 37 feet above the old streambed. According to the Corps of Engineers guidelines, a dam having between 1000 and 50,000 acre feet of storage is considered in the intermediate size range.

d. Hazard Classification - HIGH (Category I) The area downstream of the dam is a residential/urban development including residential and industrial buildings, a high school, and Interstate Route 91. If the dam were breached, there is a potential that many lives could be lost. Overtopping during a test flood (PMF) even without dam failure, yields a potential for loss of life.

e. Ownership -New Haven Water Company
Sargent Drive
New Haven, Connecticut 06506
Mr. Joseph Jiskra (203) 624-6671
Mr. Jack Reynolds

f. Purpose of Dam - Public Water Supply

g. Design and Construction History - The following information is believed to be accurate based on the plans and correspondence available, which are included in the Appendix. The dam and spillway was originally constructed in 1860-1861 by Eli Whitney and C. McClalland and Son. In 1866-1867 the spillway was raised with cemented stone masonry, 4 feet on the upstream side and 3 feet on the downstream side. In 1916 the spillway was raised with concrete to its present elevation of 36.3 and extended on the left an additional 60 feet by means of a concrete side channel. This work was engineered by Albert B. Hill and constructed by the Sperdy Engineering Co. for the New Haven Water Company. In 1964-1965, the supply and low level intake structures were rebuilt and repaired. New 42 inch steel mains were inserted into the original 48 inch steel mains and grouted in place. New intake screen facilities were constructed on the downstream side of the dam. This work was engineered by Malcolm Pirnie Engineers and constructed by C.W. Blakeslee and Sons, Inc. for the owner.

h. Normal Operational Procedures - The normal operational procedure is to provide water to the filtration plant as needed for public supply.

1.3 Pertinent Data

a. Drainage Area - 37.4 square miles. Rolling to flat and coastal terrain.

b. Discharge at Dam Site - Maximum Known Flood -Maximum water over spillway during the August and October 1955 floods was 14" on October 16,1955. Total Spillway Capacity at Elevation 41.3 (top of dam) - 9700 cfs.

c. Elevation - (Ft. above MSL, U.S.G.S. Datum)

Top of Dam:	41.3
Spillway Crest:	36.3
Streambed @ Center Line of Dam:	4.3
42" Low Level Intake	25.2
24" Feed to Filtration Plant:	25.8

d. Reservoir - Length of Normal Pool: 11,000 ft.

Length of Maximum Pool: 11,000+ ft.

e. Storage - Normal Pool: 2,720 acre ft.

At Elevation 41.3 (top of dam) 3,600 acre ft.

f. Reservoir Surface - Normal Pool: 178.3 acres

Maximum Pool: 178.3+ acres

g. Dam - Type: Downstream masonry wall with upstream earth embankment.

Length of Dam: 340 ft.

Height: 37 ft.

Top Width: 20 ft.

Side Slope:	7H to 12V upstream masonry 3H to 1V upstream earth 2H to 12V downstream masonry
Impervious Core:	Not Applicable
Cutoff:	Foundation on rock.

h. Diversion and Regulatory Tunnel - Not Applicable.

i. Spillway -

Type:	Part concrete circular (compounded) crest & sloped downstream face; part concrete ogee side channel spillway.
Length of Weir:	250 feet
Crest Elevation:	36.3
Upstream Channel:	3H to 1V

j. Regulatory Outlets - 42" and 24" Low Level Intakes

36" Feed to Filtration Plant

42" Feed to outlet channel via arch culvert.

SECTION 2: ENGINEERING DATA

2.1 Design

a. Available Data - The available data consists of drawings, correspondence and records by the State of Connecticut Water Resources Commission, the New Haven Water Company, Joseph W. Cone and others.

b. Design Features - The drawings indicate the design features stated previously herein.

c. Design Data - There were no engineering values, assumptions, test results or calculations available for the original construction or later spillway raisings. Preliminary drawings for the spillway reconstruction indicate a high water design elevation of 38.3.

2.2 Construction

a. Available Data - "As Built" drawings were available and are included in the Appendix Section B for the 1916 spillway raising. No other construction estimates or reports were available.

b. Construction Considerations - No information was available.

2.3 Operation

Lake level readings are taken daily. The maximum recorded water over the spillway was 14 inches on October 16, 1955. To our knowledge the dam spillway capacity has never been exceeded. No formal operations records exist.

2.4 Evaluation

a. Availability - Existing data was provided by the State of Connecticut and the owner. The owner made the operations available for visual inspection.

b. Adequacy - The limited amount of detailed engineering data available was generally inadequate to perform an in-depth assessment of the dam, therefore, the final assessment of this investigation must be based primarily on visual inspection, performance history, hydraulic computations of spillway capacity and approximate hydrologic assumptions.

c. Validity - A comparison of record data and visual observations reveals no observable significant discrepencies in the record data.

SECTION 3: VISUAL INSPECTION

3.1 Findings

a. General - The general appearance of the dam is good. Close inspection reveals some areas requiring maintenance.

b. Dam - The reservoir level was slightly above the spillway crest and only the upper 6 to 12 inches of stone protection of the upstream slope was exposed.

Crest - The crest of the dam is earth, and it slopes from the downstream stone wall to the upstream edge with an elevation difference of about three and a half feet. The crest is grass covered, with no cracks or erosion apparent.

Downstream Face - The downstream face is masonry with mortar and is in good condition. The only cracks observed are mostly vertical and are located at the end of the right wall of the spillway. There are no significant seeps through the wall nor wet spots downstream of the dam. Only two minor seeps were observed around two abandoned pipes through the wall.

c. Appurtenant Structures - The outlet channel for the arch culvert has stone walls which have collapsed at three locations. In the area approximately over the arch culvert, there are several depressions of the ground surface, the largest being approximately 8 feet in diameter and 2 feet deep.

The concrete retaining wall for the lateral discharge section of the spillway is in good condition.

d. Downstream Channel - The downstream channel is the natural bed of the Mill River. No obstructions to the flow are apparent near the dam. The left bank of the channel immediately downstream of the dam consists of a near vertical rock wall which rises about 175 feet from the lower stream below the dam. The bedrock consists of very hard, blue-gray aphanitic dolerite (commonly referred to as "trap rock") and is part of an extrusive flow sheet which makes up the East Rock Area.

The bedrock of the abutment exhibits well-developed columnar jointing, which is typical for a rock of this type. The rock is moderately to intensely jointed. There appear to be two (2) dominant joint patterns which intersect to form a roughly orthogonal system. The primary joint pattern strikes N5°W to N8°E and dips 60° west to 90° (vertical). A secondary pattern strikes S70°-80°E and dips 75° north to 90° (vertical).

The visual inspection indicates that the high angle jointing and weathering has resulted in occasional minor rock falls. No visual evidence was found that would suggest the possibility of major instability of the cliff. The available operating records of the dam do not contain references to major failures of the cliff since construction of the dam in 1861.

A small talus slope exists at the base of the abutment and extends out to the streambed just below the dam. The talus consists of angular fragments of dolerite which have fallen from the abutment. The fragments are generally less than 1 foot.

3.2 Evaluation

Based upon the visual inspection, it was possible to assess the dam as being generally in good condition. The following features which could influence the future stability of the dam were identified.

1. Although the stability of the dam would probably not be affected, if the ground depressions over the outlet culvert are due to loss of ground into the culvert caused by movements of its there is a danger of blockage of the culvert.

2. The upstream slope is protected against erosion with riprap up to an elevation of approximately one foot over the spillway crest. If the reservoir level were to rise over the spillway crest by more than one foot, the unprotected earthen embankment would be subject to erosion, and the resulting build-up hydrostatic pressure behind the downstream masonry wall could compromise its stability.

SECTION 4: OPERATIONAL PROCEDURES

4.1 Regulating Procedures

The operator of the dam usually regulates flow over the spillway at a level sufficient to effectively limit the amount of water in the basements of houses upstream from the dam. The general regulation plan of the reservoir consists of maintaining as much water in the reservoir as possible until the water level reaches the maximum desired height, at which point the low level lines are opened to limit the maximum water level over the spillway. Outlet valves are located both upstream and downstream of the dam; apparently, the downstream valves are normally used to regulate outflow.

4.2 Maintenance of Dam

Water level readings are taken daily. Grass on and in the vicinity of the dam is cut regulary during the growing season. Generally, on a monthly basis, the shoreline is inspected for erosion, trespassing and tree growth. Trespassing and vandalism have been consistent problems at this facility in the past.

4.3 Maintenance of Operating Facilities

All valves are opened on a monthly basis to be checked, at which time screens are back flushed and cleaned. Screen valves are also opened as needed to supply water to the filtration plant. In late summer and fall when the water level over the spillway is normally lower, the blowoff is opened and any debris, logs, etc. are removed from the spillway and low level intakes to prevent obstruction.

4.4 Description of any Warning System in Effect

No formal warning system is in effect at this time. The New Haven Water Company office would be notified should any emergency situations arise.

4.5 Evaluation

The operation and maintenance procedures are generally satisfactory, however, there are some areas requiring improvement. A formal program of operation and maintenance procedures should be implemented, including documentation to provide complete records for future reference. Also, a formal warning system should be developed and implemented within the time frame indicated in Section 7.3b. Remedial operation and maintenance recommendations are presented in Section 7.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 Evaluation of Features

a. Design Data - No computations could be found for the original dam construction or later raisings.

b. Experience Data - No information on serious problem situations arising at the dam were found, and it does not appear the dam has been overtopped. The maximum height of water over the spillway during the floods of August and October 1955 was 14 inches on October 16, 1955.

c. Visual Observations - Upstream problems of flooded basements occur when water level over the spillway exceeds 12-14 inches. Runoff will increase in the future, as the area upstream of the dam is an urban and developing area. Significant amounts of debris have collected at the top of the spillway. As the spillway is wide and not spanned by a bridge, the possibility of the spillway being obstructed is minimal.

d. Overtopping Potential - The test flood for this high hazard intermediate size dam is equivalent to the Probable Maximum Flood (PMF) of 46,500 cubic feet per second (cfs).

Based upon our hydraulics computations, the spillway capacity is 9700 cfs. Based upon "Preliminary Guidance for Estimating Maximum Probable Discharges", dated March 1978, peak inflow to the reservoir is 48,600 cfs (Appendix D-7); peak outflow (Test Flood) is 46,500 cfs with the dam overtopped 4.4 feet (Appendix D-12).

e. Spillway Adequacy - The spillway will pass approximately 21 percent of the 46,500 cfs Test Flood.

SECTION 6: STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations - There exists a depressed area on the top of the dam to the immediate right of the spillway and above the masonry abutment which could cause ponding of water on the dam and subsequent seepage into the masonry. Should freezing occur, the result could be deterioration and/or movement of the masonry. Some cracking in the masonry abutment was observed during the field inspection.

b. Design and Construction Data - The design and construction data available do not include information concerning the cross section of the dam, and thus it is not possible to analyze its stability. There is no design data available to indicate a stability or seepage analysis was performed. Past history of the dam indicates it has performed adequately. It is possible that long term future stability of the dam could be affected by deterioration of the masonry due to increased seepage.

c. Operating Records - The operating records do not include any indication of dam instability since its construction in 1861, or since subsequent modifications were performed.

d. Post Construction Changes - The post construction changes consisted of raising the spillway and dam in 1867 and 1916 and changes in the gate houses, screen chamber and piping in 1964-1965. These latter changes involved some raising of the grade immediately downstream of the dam, and thus did not decrease the degree of stability of the dam.

e. Seismic Stability - The dam is located in Seismic Zone 1 in accordance with the seismic risk map in the USCE guidelines for the dam inspection, and in accordance with the guidelines, it need not be analyzed for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition - Based upon the visual inspection at the site and past performance history, the dam is considered to be in good condition. No evidence of structural instability in the masonry or the earth embankment was observed, and the condition of the embankment and the masonry is generally good. There are some areas which require attention, including the arch culvert outlet structure, the availability of sufficient freeboard protected against erosion, and required maintenance of the outlet channel.

Based upon our hydraulics computations, the spillway capacity is 9700 cubic feet per second, which is equivalent to approximately 21 percent of the Test Flood. Based upon "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March 1978, peak inflow to the reservoir is 48,600 cubic feet per second; peak outflow is 46,500 cubic feet per second with the dam overtopped 4.4 feet.

Utilizing the April 1978 "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", the peak failure outflow from the dam breaching would be 44,800 cubic feet per second. The dam is located upstream of industrial buildings, a high school and residential/urban developments in the New Haven area. A breach of the dam would develop an 18 foot wave and would create flooding downsteam of the dam causing severe damage to life and property.

b. Adequacy of Information - There is no data available on the design and construction of the masonry dam and its upstream earth embankment. Thus the evaluation of dam stability is based soley on visual inspection and operational records.

c. Urgency - It is recommended that the measures presented below be implemented within the time frame indicated in Sections 7.2 and 7.3.

d. Need for Additional Information - There is a need for more information as recommended below in Section 7.2.

7.2 Recommendations

The following recommendations should be instituted within one year of the owner's receipt of this report.

1. Based upon the rough computation in Appendix D, the dam spillway capacity will be exceeded by the Test Flood. More sophisticated flood routing should be undertaken by hydrologists/hydraulics engineers to refine the Test Flood figures. A study should be undertaken and recommendations made to increase the spillway capacity to an acceptable level based upon the refined Test Flood figures.

2. The integrity of the arch culvert outlet structure should be examined, particularly in those areas corresponding to the depressions of the ground surface, to assure that the culvert remains clear of debris.

7.3 <u>Remedial Measures</u>

a. <u>Alternatives</u> - This study has identified no practical alternatives to the above recommendations.

b. <u>Operation and Maintenance Procedures</u> - The following measures should be undertaken within one year of the owner's receipt of this report, and continued on a regular basis.

1. Round-the-clock surveillance should be provided by the owner during periods of unusally heavy precipitation. The owner should develop a formal warning system with local officials for alerting downstream residents in case of an emergency.

2. The spillway modifications required should be implemented based upon the spillway capacity recommendations of the study above, in Section 7.2.1.

3. The stone walls of the outlet channel for the low level outlet pipes, should be repaired where collapse and subsequent erosion have occurred.

4. The depressed area on the top of the dam should be filled to prevent ponding of water and seepage into the masonry. Cracks in the masonry abutment to the right of the spillway should be monitored regularly for any worsening.

5. The slight seepage in the abandoned outlet works should be monitored regularly for any increase in the rate of seepage.

6. During the course of this study, it was brought to our attention that the New Haven Water Company instituted a yearly program for inspection of all their dams, including Lake Whitney Dam, by a consultant competent in the field of dam inspection. This program which has been in effect for the past two years, is comendable and should be continued in the future.

7. The present valve inspection and maintenance program should be continued. This, and all other inspection programs of the dam, should be accurately documented in both procedure and inspection results for future reference.

APPENDIX

SECTION A: VISUAL OBSERVATIONS

VISUAL INSPECTION CHECK LIST
PARTY ORGANIZATION

PROJECT Lake Whitney Dam

DATE: May 30, 1978

TIME: 8:30 a.m.

WEATHER: Clear, hot

W.S. ELEV. 33.3 U.S. 8.0 DN.S

PARTY:	INITIALS:	DISCIPLINE:
1. Mike Horton	MH	Structural
2. Hector Moreno	HM	Hydraulics
3. Gonzalo Castro	GC	Geotechnical
4. Dean Thomasson	DT	Party Chief
5.		
6.		

PROJECT FEATURE	INSPECTED BY	REMARKS
1. Masonry Dam Embankment	GC/MH	
2. Spillway-Approach, Channel, Weir, Discharge Channel	GC/MH	
3. Outlet Works-Inlet Channel and Inlet Structure	MH	
4. Outlet Works-Control Tower, Operating House, Gate Shafts	MH	
5. Outlet Works-Outlet Structure and Outlet Channel	GC/MH	
6. Outlet Works-Service Bridge (Pedestrain/Vehicular)	MH	
7. Reservoir	HM/DT	
8. Operation and Maintenance	HM/DT	
9. Safety and Performance Instrumentation	DT	
10.		
11.		
12.		

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Whitney Dam **DATE** May 30, 1978

PROJECT FEATURE Masonry Dam Embankment

AREA EVALUATED	BY	CONDITION
Crest Elevation		
Current Pool Elevation		
Maximum Impoundment to Date		
Surface Cracks	GC	Vertical cracks at end of right wall of spillway.
Pavement Condition	GC	No pavement.
Movement or Settlement of Crest	GC/MH	None apparent. Hole in crest above abutment. Possible drainage problem.
Lateral Movement	GC	None apparent.
Vertical Alignment	GC	No misalignment observable.
Horizontal Alignment	GC	No misalignment observable.
Condition at Abutment and at Masonry Structures	MH	Crack in abutment.
Indications of Movement of Structural Items on Slopes	GC	None observed.
Trespassing of Slopes	GC	None observed.
Sloughing or Erosion of Slopes or Abutments	GC	None observed.
Rock Slope Protection-Riprap Failures	GC	Riprap under water, could not be observed.
Unusual Movement or Cracking at or near Toe	GC	None observed.
Unusual Embankment or Downstream Seepage	GC	None observed.
Piping or Boils	GC	None observed.
Foundation Drainage Features	GC	None known.
Toe Drains	GC	A4-in. tile drain at toe as per Dwg. MP-33-1, but not observable. Flow could not be observed as it leads to a covered "cobble apron" or dry well.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Whitney Dam **DATE** May 30, 1978

PROJECT FEATURE Masonry Dam Embankment

AREA EVALUATED	BY	CONDITION
Instrumentation Systems	GC	None known.
Other Observations	GC	The upstream stone masonry wall does not reach the crest elevation of 38 but only to about 34, and thus the top 4 ft. of freeboard are not protected.
	GC	Depressions noted D.S. of dam as noted in plan. Largest depression is about 8 ft. in diameter and 2 ft. deep. They correspond to location of existing arch conduit and could be due to loss of ground into the conduit.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Whitney Dam **DATE** May 30, 1978

PROJECT FEATURE Spillway-Approach, Channel, Weir, Discharge Channel

AREA EVALUATED	BY	CONDITION
a. Approach Channel	GC	None could be observed, reservoir full.
General Condition		
Loose Rock Overhanging Channel		
Trees Overhanging Channel	MH	None.
Floor of Approach Channel		
b. Weir and Training or Sidewalls		
General Condition of Concrete	GC	Good.
Rust or Staining	GC	None.
Spalling	GC	None.
Any Visible Reinforcing	GC	None.
Any Seepage or Efflorescence	GC	One efflorescense spot a few inches in size on retaining wall on left abutment.
Drain Holes	GC	No drainage holes observed
c. Discharge Channel		
General Condition	GC	Good.
Loose Rock Overhanging Channel	GC	Steep rock cliff at left abutment with possibility of some rock falls which would not be critical.
Trees Overhanging Channel	GC	None near dam.
Floor of Channel	GC	Natural gravelly soil.
Other Obstructions	GC	None.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Whitney Dam **DATE** May 30, 1978

PROJECT FEATURE Outlet Works-Inlet Channel & Inlet Structure

AREA EVALUATED	BY	CONDITION
a. Approach Channel		
Slope Conditions		
Bottom Conditions		
Rock Slides or Falls		
Log Boom		
Debris		
Condition of Concrete Lining		
Drains or Weep Holes		
b. Intake Structure		
Condition of Concrete	MH	Good.
Stop Logs and Slots		

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Whitney Dam **DATE** May 30, 1978

PROJECT FEATURE Outlet Works-Control Tower, Operating House, Gate Shafts

AREA EVALUATED	BY	CONDITION
a. Concrete and Structural		
General Condition	MH	Good.
Condition of Joints	MH	One joint seeping.
Spalling	MH	None.
Visible Reinforcing	MH	None.
Rusting or Staining of Concrète	MH	None.
Any Seepage or Efflorescence	MH	Little.
Joint Alignment		
Unusual Seepage or Leaks in Gate Chamber		
Cracks		
Rusting or Corrosion of Steel		
b. Mechanical and Electrical		
Air Vents		
Float Wells		
Crane Hoist		
Elevator		
Hydraulic System		
Service Gates		
Emergency Gates		
Lighting Protection System		
Emergency Power System		

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Whitney Dam **DATE** May 30, 1978

PROJECT FEATURE Outlet Works-Outlet Structure and Outlet Channel

AREA EVALUATED	BY	CONDITION
General Condition of Concrete		
Rust or Staining		
Spalling		
Erosion or Cavitation		
Visible Reinforcing		
Any Seepage or Efflorescence		
Condition at Joints		
Drain Holes	GC	None observed.
Channel		
Loose Rock or Trees Overhanging Channel	GC	None observed.
Condition of Discharge Channel	GC	At three locations sections of the stone wall are missing.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Whitney Dam **DATE** May 30, 1978

PROJECT FEATURE Outlet Works-Service Bridge (Pedestrian/Vehicular)

AREA EVALUATED	BY	CONDITION
a. Super Structure	MH	NA.
Bearings	MH	NA.
Anchor Bolts	MH	NA.
Bridge Seat	MH	NA.
Longitudinal Members	MH	Good.
Under Side of Deck	MH	Good.
Secondary Bracing	MH	NA.
Deck	MH	Good.
Drainage System	MH	NA.
Railings	MH	Post bases split due to freeze thaw action.
Expansion Joints		
Paint	MH	None.
b. Abutment & Piers	MH	NA.
General Condition of Concrete	MH	NA.
Alignment of Abutment	MH	NA.
Approach to Bridge	MH	NA.
Condition of Seat & Backwall	MH	NA.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Whitney Dam **DATE** May 30, 1978

PROJECT FEATURE Reservior

AREA EVALUATED	BY	CONDITION
Shoreline	DT	Deciduous vegetation. Residential area.
Sedimentation	DT	Only near new construction.
Potential Upstream Hazard Areas	DT	Flooded basements when water 12" to 14" over spillway.
Watershed Alteration-Runoff Potential	DT	High-developing urban area.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Whitney Dam **DATE** May 30, 1978

PROJECT FEATURE Operations and Maintenance

AREA EVALUATED	BY	CONDITION
a. Reservoir Regulation Plan		
Normal Conditions	DT	Maintain as much water in reservoir as possible up to 12"-14".
Emergency Plans	DT	No formal procedure.
Warning System	DT	None known.
b. Maintenance (Type) (Regularity)		
Dam	DT	As needed. Shoreline inspected monthly
Spillway	DT	Blowoff opened and both blowoff and spillway cleared of logs, debris, etc. at times of low water (fall & spring)
Outlet Works	DT	Inspected monthly and maintained as needed. Valves opened monthly.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Whitney Dam **DATE** May 30, 1978

PROJECT FEATURE Safety and Performance Instrumentation

AREA EVALUATED	BY	CONDITION
Headwater and Tailwater Gages	PH	Water level readings daily.
Horizontal and Vertical Alignment Instrumentation (Concrete Structures)	PH	NA.
Horizontal and Vertical Movement, Consolidation, and Pore-Water Pressure Instrumentation (Embankment Structures)	PH	None.
Uplift Instrumentation	PH	None.
Drainage System Instrumentation	PH	None.
Seismic Instrumentation	DT	None.

APPENDIX
SECTION B: EXISTING DATA

SPECIAL NOTE

SECTION B

AVAILABILITY OF DATA

The correspondence listed in the summary of contents and the plans listed in the Table of Contents, Appendix Section B, are included in the master copy of this report, which is on file at the office of the Army Corps of Engineers, New England Division, in Waltham, Massachusetts.

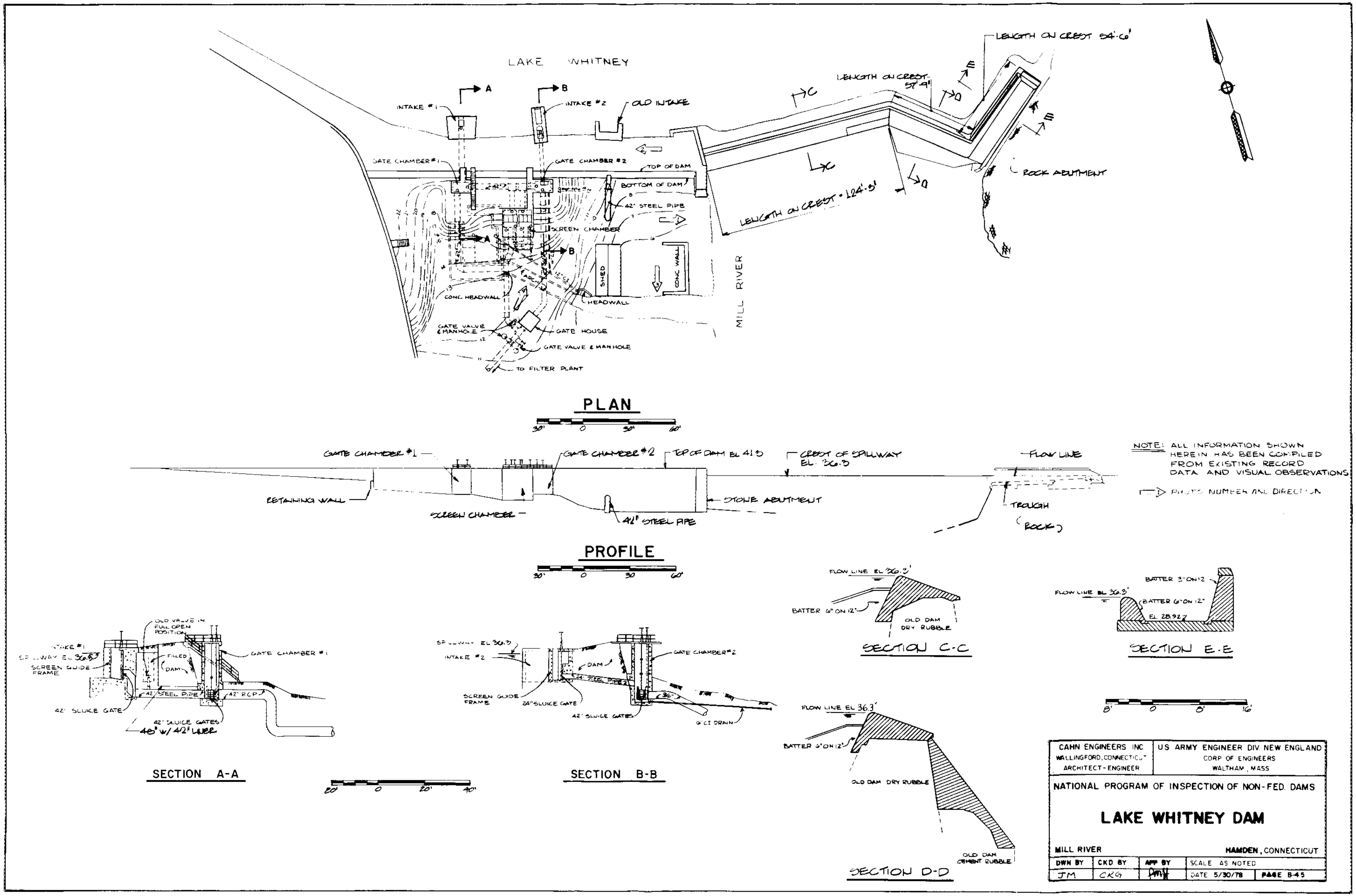
SECTION B: EXISTING DATA

SUMMARY OF CONTENTS

DATE	TO	FROM	SUBJECT	PAGE
Apr. 29, 1963	A.L. Corbin, Jr.	Joseph A. Novaro Chief Engineer, New Haven Water Company[2]	West River Watershed	B-1
July 30, 1963	Files	Water Resources Commission[1]	Dam Inventory Data	B-4
Apr. 30, 1965	Joseph W. Cone	New Haven Water Company[2]	Transmittal of (and including) lake level and rain gauge records.	B-6
July 24, 1965	William P. Sander Water Resources Commission	Joseph W. Cone[1]	Corrections on dams owned by New Haven Water Company and additional enclosures.	B-13
July 15, 1966	William Wise, Dir., Water Resources Commission	Joseph A. Novaro[2]	Progress Report for West River System Studies.	B-19
August 1974	Files	New Haven Water Company[2]	Whitney Dam data sheets, map and photographs.	B-20

[1]Obtained from State of Connecticut Water Resources Commission

[2]Obtained from New Haven Water Company



CAHN ENGINEERS INC WALLINGFORD, CONNECTICUT ARCHITECT - ENGINEER	US ARMY ENGINEER DIV NEW ENGLAND CORP OF ENGINEERS WALTHAM, MASS

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LAKE WHITNEY DAM

MILL RIVER — HAMDEN, CONNECTICUT

DWN BY	CKD BY	APP BY	SCALE AS NOTED	
JM	CKG	[illegible]	DATE 5/30/78	PAGE B-45

APPENDIX

SECTION C: DETAIL PHOTOGRAPHS

PHOTO NO.1 - Spillway and natural rock abutment to left of dam.

PHOTO NO.2 - General view of crest of dam to right of spillway showing three intake structures.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	LAKE WHITNEY DAM MILL RIVER HAMDEN, CONNECTICUT
CAHN ENGINEERS INC. WALLINGFORD, CONN. ARCHITECT—— ENGINEER		CE # 27 531 GF DATE 5/30/78 PAGE C-1

PHOTO NO.3 - Deterioration of stone wall lining the outlet channel.

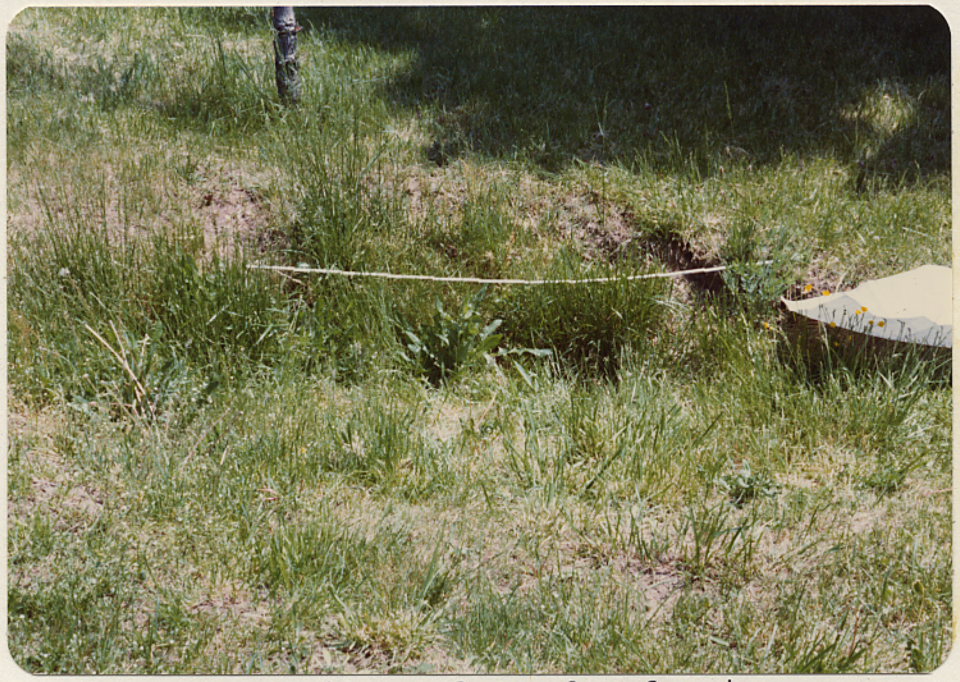


PHOTO NO.4 - Subsidence of ground surface in area over arch culvert. Note six (6) foot rule and unfolded plan.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	LAKE WHITNEY DAM MILL RIVER HAMDEN, CONNECTICUT
CAHN ENGINEERS INC. WALLINGFORD, CONN. ARCHITECT—ENGINEER		CE # 27 531 GF DATE 5/30/78 PAGE C-2

APPENDIX

SECTION D: HYDRAULIC/HYDROLOGIC COMPUTATIONS

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

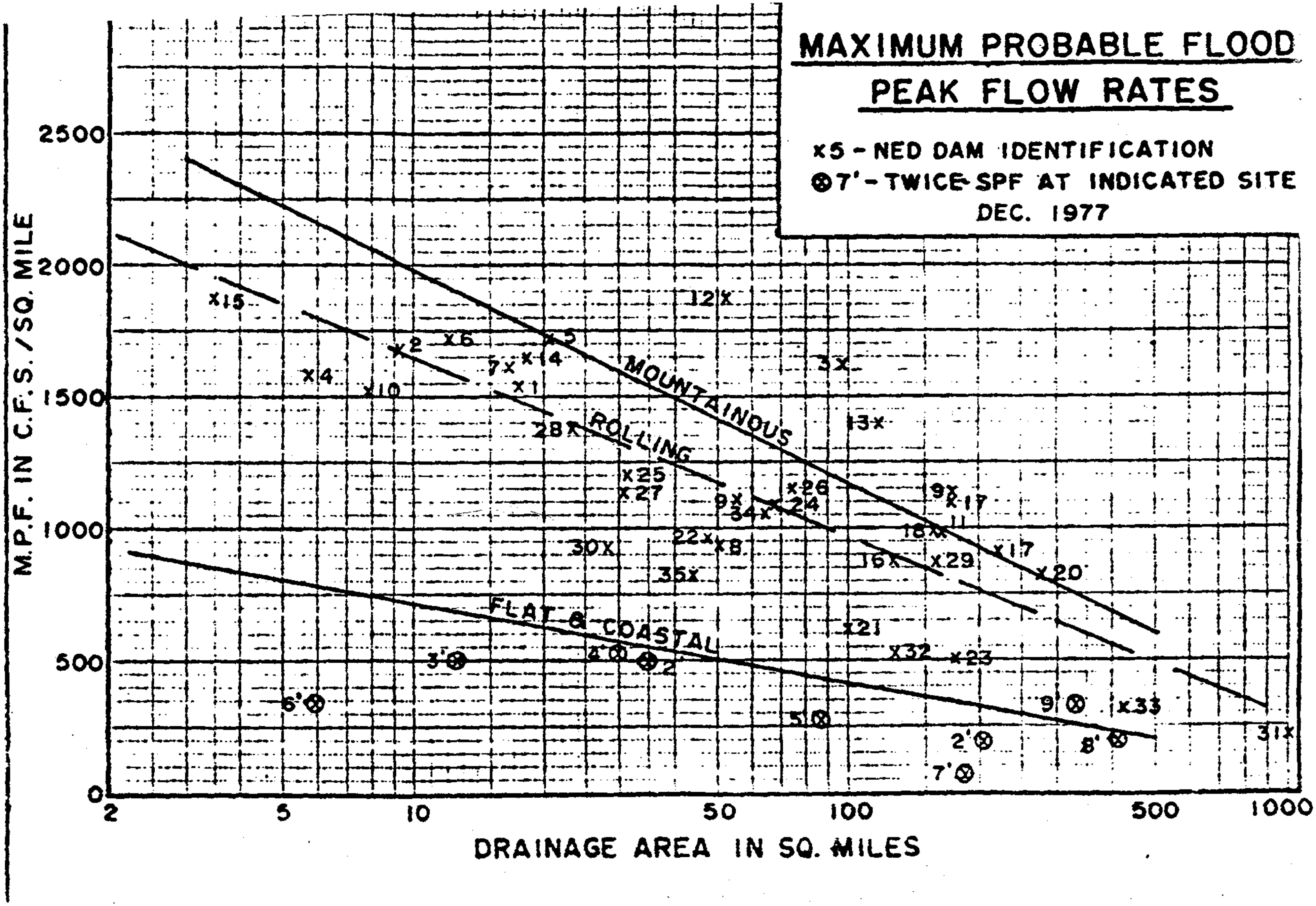
March 1978

MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

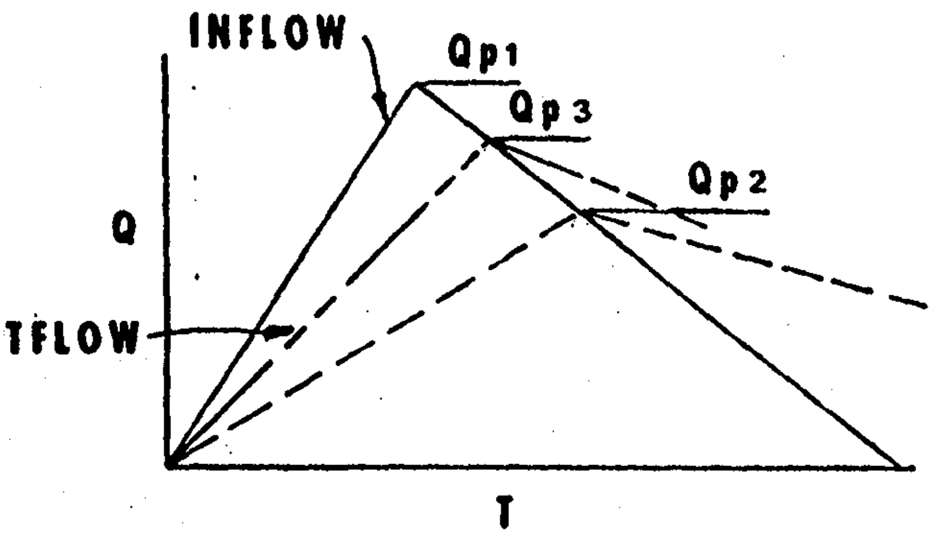
	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

MAXIMUM PROBABLE FLOWS BASED ON TWICE THE STANDARD PROJECT FLOOD (Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330


MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITE
DEC. 1977
M.P.F. IN C.F.S./SQ. MILE
DRAINAGE AREA IN SQ. MILES
MOUNTAINOUS
ROLLING
FLAT & COASTAL
2500
2000
1500
1000
500
0
2
5
10
50
100
500
1000

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Q_{p1}) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Q_{p1}".

b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

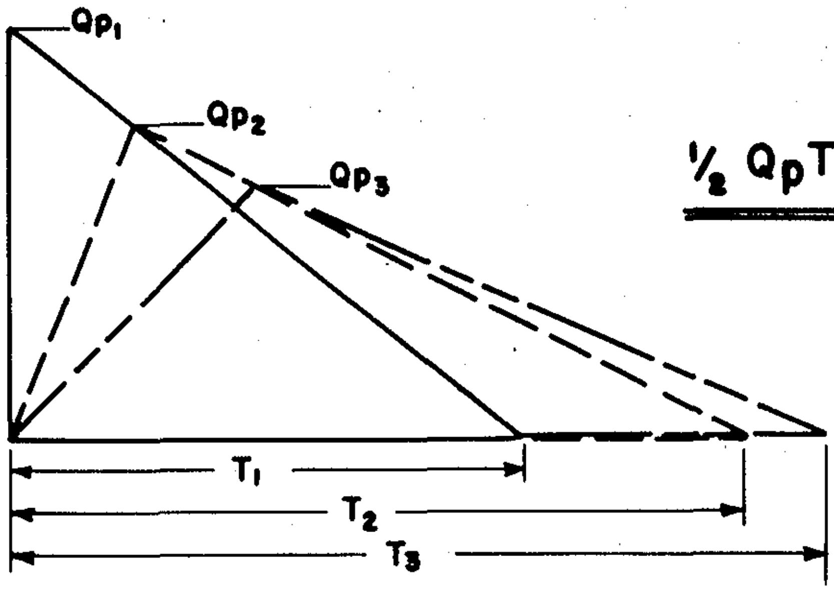
c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Q_{p2}"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Q_{p3}".

"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$$\tfrac{1}{2} Q_p T = 12 S$$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27} W_b \sqrt{g}\ Y_o^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(\text{TRIAL}) = Qp_1 \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1 \left(1 - \frac{V_{ave}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

HYDROLOGIC / HYDRAULIC INSPECTION

WHITNEY LAKE, HAMDEN, CT. - DOWNSTREAM FLOOD HAZARD

1) ESTIMATE OF DOWNSTREAM FAILURE HYDROGRAPHS
(SEE ACE "RULE OF THUMB" GUIDANCE FOR ESTIMATING THESE HYDROGRAPHS)

a) ESTIMATE OF RESERVOIR STORAGE AT TIME OF FAILURE

AVERAGE SURCHARGE WL. ELEV. 45.7' MSL (SEE HM ESTIMATE 5/18/78 p.6.

RESERVOIR AREA @ FLOW LINE : $A_{FL} = 178.3$ AC. (ELEV.* 36.3' MSL)
*NOTE: DWGS SHOW ELEVS. IN NEW HAVEN DATUM (MHW) - MSL (USGS) ≡ MHW + 3.3'

i) FROM NEW HAVEN WATER CO. DATA ON WHITNEY DAM (AUG. 1974)
(EL. 27.4 MHW TO EL. 33 MHW)
CAPACITY OF TOP 5.6' ONLY, TO FLOW LINE (EL. 30.7 MSL TO EL. 36.3 MSL
IS USABLE CAPACITY OF RESERVOIR = 258 MG. ≡ 792 AC-FT

ii) FROM INVENTORY OF DAMS IN THE U.S. (ORIGINAL RES. - 1861) IMPOUNDING CAPACITIES:

MAXIMUM : 2140 AC-FT.
NORMAL : 1926 AC-FT
$\Delta S = 214$ AC-FT.

ORIGINAL SPILLWAY ELEV. 27± MHW ≡ 30.3± MSL (ASSUMED TO CORRESPOND TO "NORMAL" CAPACITY SHOWN ABOVE) ∴

iii) ESTIMATED CAPACITY TO PRESENT FLOW LINE (SPILLW. CREST ELEV. 36.3 MSL)

$S_{FL} \doteq 1930 + 790 = 2720$ AC-FT (TO ELEV. 36.3 MSL)

iv) ESTIMATED RES. STORAGE (S) AT TIME OF FAILURE (TO EL. 45.7 MSL)

$S \doteq 2720 + 180 \times 9.4 = 4410 \approx 4400$ AC-FT. ∴ $\frac{S}{2} = 2200$ AC-FT

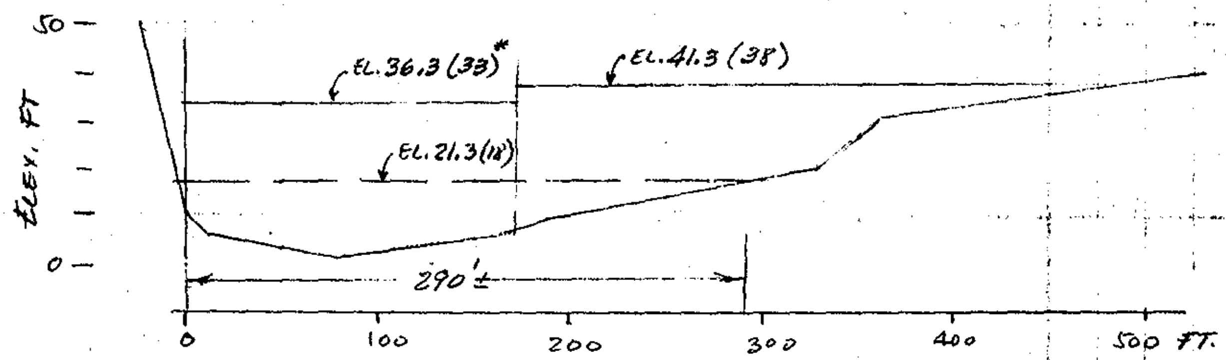
HYDROLOGIC / HYDRAULIC INSPECTION

WHITNEY LAKE, HAMDEN, CT - DOWNSTREAM FLOOD HAZARD

1-Cont'd) ESTIMATE OF D/S FAILURE HYDROGRAPHS.

b) PEAK FAILURE OUTFLOW (Q_{p_1})

i) BREACH WIDTH



APPROX. CROSS SECTION AT DAM

*NOTE: ELS. ARE MSL AND (MHW)

∴ 40% OF DAM LENGTH ACCROSS RIVER AT MID HEIGHT :

$$W = 290 \times 0.4 = 116'$$

∴ ASSUME BREACH WIDTH : $W_b \simeq \underline{100'}$

ii) TOTAL HEIGHT AT FAILURE : $Y_0 = 45.7 - 4.3 = 41.4'$

(BOTTOM OF STREAM BED AT ± EL. 1' MHW = ± EL 4.3 MSL)

iii) PEAK FAILURE OUTFLOW :

$$Q_{p_1} = \frac{8}{27} W_b \sqrt{g}\, Y_0^{3/2} = \underline{\underline{44800}} \text{ CFS}$$

Cahn Engineers Inc. Consulting Engineers

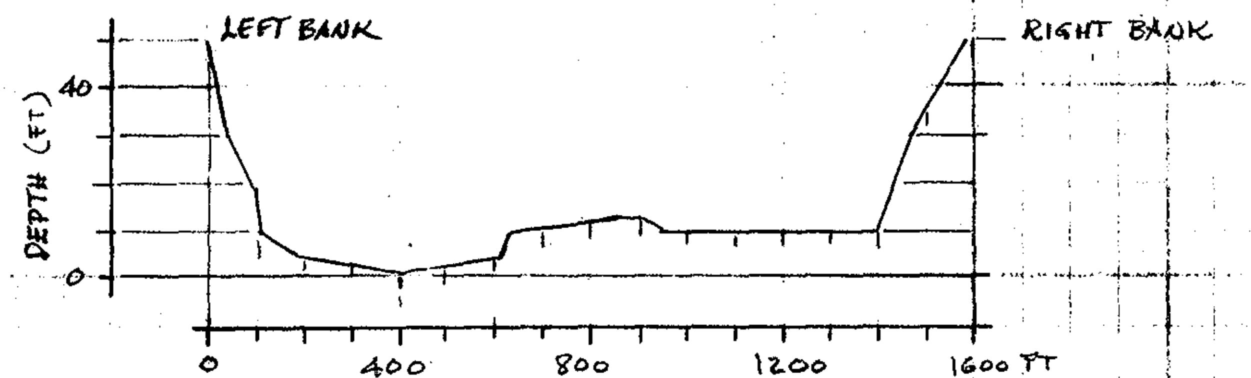
Project INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND — Sheet 3 of 4
Computed By [illegible] — Checked By D.[illegible] — Date 5/18/78
Field Book Ref. — Other Refs. CE #27-531-GF — Revisions

HYDROLOGIC/HYDRAULIC INSPECTION

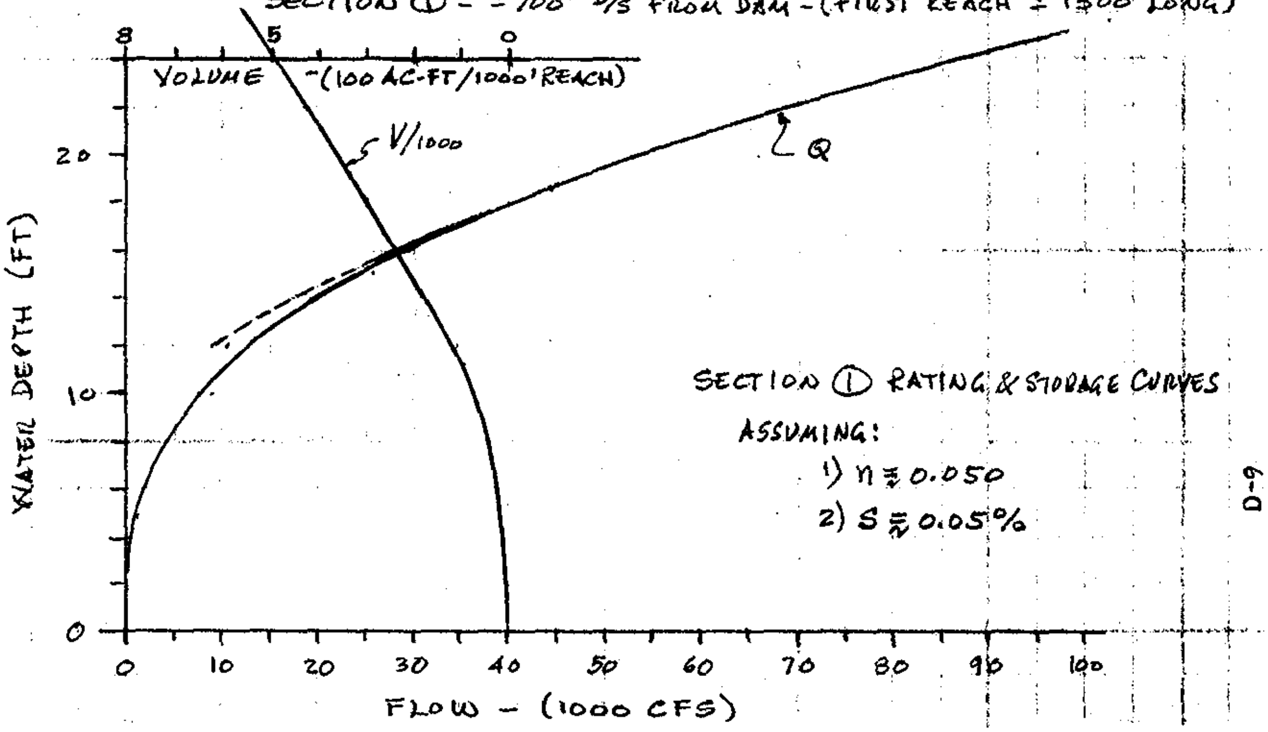
WHITNEY LAKE, HAMDEN, CT. - DOWNSTREAM FLOOD HAZARD

1-Cont'd) ESTIMATE OF D/S FAILURE HYDROGRAPHS

c) TYPICAL CROSS-SECTIONS & RATING CURVES
(FROM TOPOGRAPHIC MAP OF NEW HAVEN - 1965 - SCALE 1" = 600')



SECTION ① - ±700' D/S FROM DAM - (FIRST REACH ± 1300' LONG)



SECTION ① RATING & STORAGE CURVES
ASSUMING:
1) n ≅ 0.050
2) S ≅ 0.05%

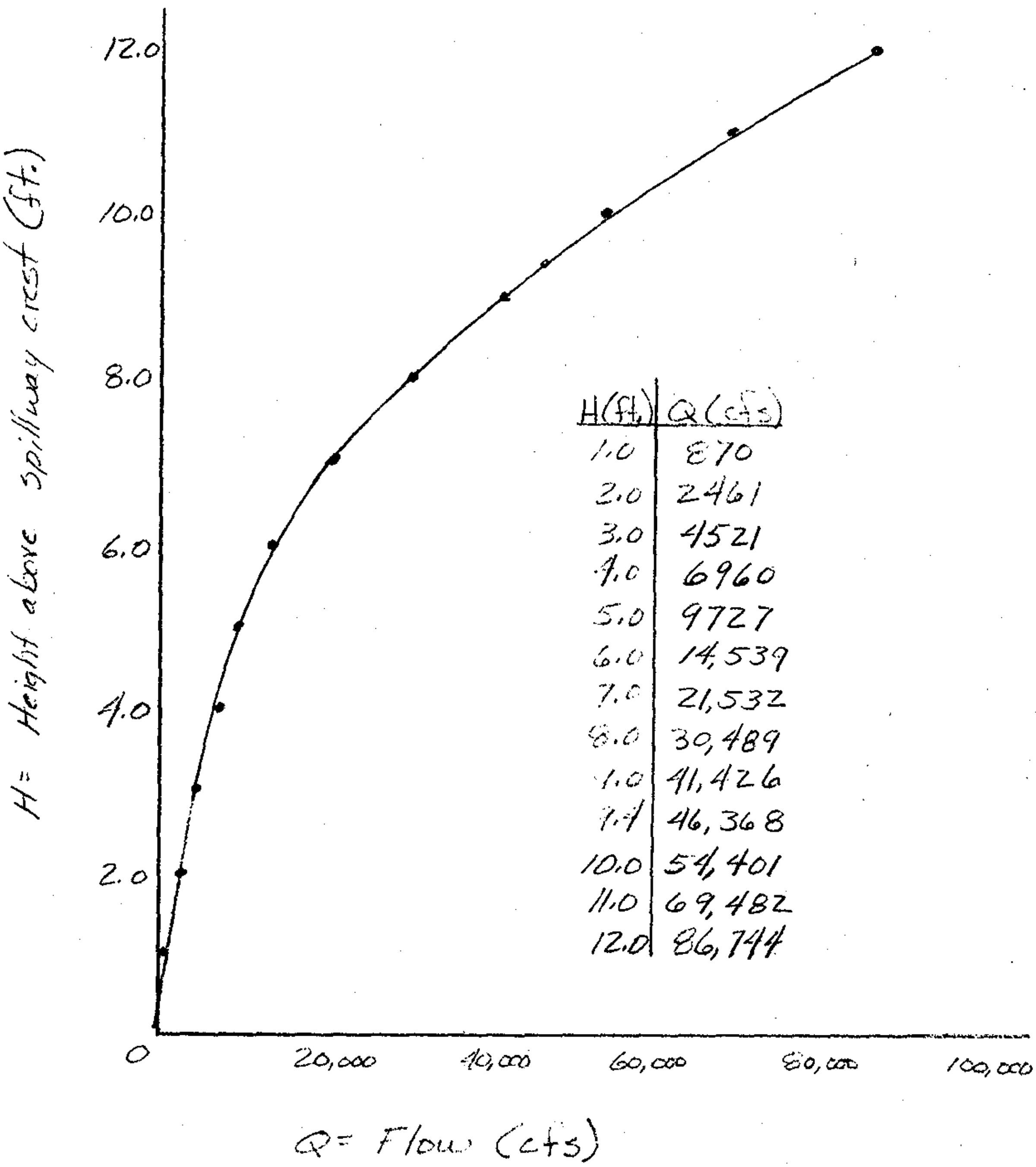
D-9

Cahn Engineers Inc. **Consulting Engineers**

oject: LAKE WHITNEY DAM — Sheet 1 of 1
mputed By: HM/CRG — Checked By: — Date: 9/8/78
ld Book Ref.: — Other Refs.: — Revisions:

SPILLWAY RATING CURVE

$$Q = 870H^{3/2} + 1590(H_1 - 5)^{3/2} + 163(H-5)^{5/2}$$



H(ft.)	Q (cfs)
1.0	870
2.0	2461
3.0	4521
4.0	6960
5.0	9727
6.0	14,539
7.0	21,532
8.0	30,489
9.0	41,426
9.4	46,368
10.0	54,401
11.0	69,482
12.0	86,744

D-9A

HYDROLOGIC / HYDRAULIC INSPECTION

WHITNEY LAKE, HAMDEN, CT. - D/S FLOOD HAZARD

1-Cont'd) ESTIMATE OF D/S FAILURE HYDROGRAPHS

d) REACH OUTFLOW (Q_{P_2})

i) @ $Q_{P_1} = 44800$ FROM RATING CURVE - STAGE $\cong$ 18.5

$\therefore$ VOLUME IN REACH : $V_1 = 320 \times 1.3 = 416 \cong 420^{AC\text{-}FT} < \frac{1}{2} S$ ok
(S = 4400 AC-FT)

ii) Q_{P_2} :

$$Q_{P_2}(\text{TRIAL}) = Q_{P_1}\left(1 - \frac{V_1}{S}\right) = 44800\left(1 - \frac{420}{4400}\right) = 40500^{CFS.}$$

iii) @ $Q_{P_2} = 40500$ STAGE $\cong$ 18' $V_2 = 300 \times 1.3 \cong 390^{AC\,FT}$

iv) AVE. VOLUME IN REACH : $V_{AVE} = 405^{AC\,FT}$

$$\therefore \; Q_{P_2} = 44800\left(1 - \frac{405}{4400}\right) = \underline{\underline{40700}}^{CFS.} \quad \text{STAGE} \cong \underline{18'}$$

NOTE: ACTUALLY THE PEAK FAILURE OUTFLOW, DEPENDING OF THE LOCATION OF THE BREACH COULD BE AS LARGE AS ± 84000 CFS WHEN COMBINED WITH THE OUTFLOW FROM SURCHARGE OVER THE UNBREACHED PORTIONS OF THE DAM. THIS WILL CORRESPOND THEORETICALLY TO A STAGE OF ±24'. THE EFFECT OF THE FAILURE OUTFLOW THRU THE BREACH AND THE CORRESPONDING STAGE (±18') IS HOWEVER OF SUCH A MAGNITUDE AS TO MAKE UNWARRANTED ANY FURTHER SPECULATION.

THEORETICALLY, THE APPROX. HEIGHT OF THE WAVE IMMEDIATELY D/S OF DAM SITE IS (±) $Y \cong 0.44 Y_0 = 0.44 \times 41.4 \cong \underline{18'}$

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Project INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND Sheet 1 of 6
Computed By HLL Checked By R. SAHN Date 5/12/78
Field Book Ref. Other Refs. CE#27-531-GF Revisions

HYDROLOGIC/HYDRAULIC INSPECTION

WHITNEY LAKE, HAMDEN, CT.

1) MAXIMUM PROBABLE FLOOD - PEAK FLOOD RATE

a) WATERSHED CLASSIFIED AS "ROLLING" TO "FLAT & COASTAL"

∴ USE MPF "ROLLING" CURVE FROM THE GUIDE CURVES FURNISHED BY THE ACE NEW ENGLAND DIV. OFFICE FOR DETERMINATION OF PMF.

b) WATERSHED AREA: D.A. = 37.4 SQ MI (NEW HAVEN WATER CO. DATA, AUG. 1974)

NOTE: OUR D.A. CHECK GIVES 36.2 SQ. MI.

c) FROM GUIDE CURVES:

MPF ≅ 1300 CFS/SQ MI.

d) MPF - PEAK INFLOW

Q = 1300 x 37.4 ≅ 48600 CFS (Q = 47000 CFS CE. D.A.)

2) SPILLWAY DESIGN FLOOD (SDF)

a) CLASSIFICATION OF DAM ACCORDING TO ACE RECOMM. GUIDELINES:

i) SIZE (IMPOUNDMENT): STORAGE (MAX) = 2140 AC.FT.* - (INTERM.)

HEIGHT = 37 FT. - (SMALL)

∴ SIZE CLASSIFICATION OF DAM: "INTERMEDIATE" SIZE

*NOTE: STORAGE, FROM INVENTORY OF DAMS IN THE U.S. (3/10/78 P.16) - (ORIGINAL DAM) - PRESENTLY, STORAGE (MAX) IS ESTIMATED AT (±) 3600 AC-FT. HEIGHT, FROM NEW HAVEN WATER CO. DRAWINGS (1914)

HYDROLOGIC / HYDRAULIC INSPECTION

WHITNEY LAKE, HAMDEN, CT.

2, a - Cnt'd) SDF - CLASSIFICATION OF DAM.

ii) HAZARD POTENTIAL:

WHITNEY DAM IS U/S INDUSTRIAL BUILDINGS, HIGH SCHOOL, RTE I-91, URBAN DEVELOPMENTS IN NEW HAVEN / WATERVILLE AREA.

THEREFORE IS CLASSIFIED AS "HIGH" HAZARD POTENTIAL.

iii) SDF:

FROM ACE RECOMMENDED GUIDELINES, FOR INTERMEDIATE HIGH HAZARD POTENTIAL DAMS.

$$SDF = MPF \approx 48600^{cfs}$$

3) EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES.

NOTE: ESTIMATE MADE IN ACCORDANCE WITH PROCEDURE OUTLINED IN ACE-NEW ENGLAND DIV. GUIDELINE SHEET.

a) PEAK INFLOW (SDF = MPF): $Q_{P_1} = 48600^{cfs}$

HYDROLOGIC/HYDRAULIC INSPECTION

WHITNEY LAKE, HAMDEN, CT.

3-Cont'd) EFFECT OF SURCHARGE STORAGE ON MPD'S

b) SURCHARGE HEIGHT TO PASS Q_{P_1} :

SEE SHEETS ON DATA AND SPILLWAY DISCHARGE COEFF. ASSUMPTIONS (HM 5/9/78).-

SPILLWAY DISCHARGE AT HIGH HEADS IS ASSUMED TO BE ABOUT:

$$Q \simeq 870 H^{3/2} \qquad H \simeq \left(\frac{Q}{870}\right)^{2/3}$$

$\therefore$ @ $Q_{P_1} = 48600^{cfs}$ THEOR. $H_1 \simeq 14.6' > 5'$

NOTE: TOP OF DAM ABOVE SPILLWAY CREST $H = 5'$
SPILLWAY CAPACITY $(H=5')$ $Q \simeq 9700^{CFS}$
SO DAM WILL BE OVERTOPPED $\therefore$ DETERMINE ACTUAL H_1

i) TOP OF DAM LENGTH = ± 250' (w/o SPILLWAY) PLUS ± 280'*
BERM OR LOW WALL ALONG WHITNEY AVE. $\therefore$ $L_D \simeq 530'$
TOP WIDTH $\simeq$ 20' (NEW HAVEN WATER Co. DATA, AUG. 1974)
ASSUME DISCH. COEFF. (OVERTOPPING) $C_D \simeq 3.0$

$$\therefore C_D L_D = \underline{1590}$$

SPILLWAY + DAM APPROX. DISCHARGE BASED ON HEAD (APPROX) ABOVE SPILLWAY (H_S):
HEAD OVER DAM = $H_S - 5$ (FEET)

*FIELD OBSERVATION

HYDROLOGIC / HYDRAULIC INSPECTION

WHITNEY LAKE, HAMDEN, CT.

3 - CONT'D) EFFECT OF SURCHARGE STORAGE ON MPD'S.

ii) APPROX. SPILLW. / DAM RATING EQUATION:

$$Q_{SD} = 870 H^{3/2} + 1590 (H-5)^{3/2}$$

iii) GROUND AT RIGHT BANK FORMS A BERM ALONG WHITNEY AVE. SLOPING GRADUALLY TO (±) ELEV. 45.5' MSL IN (±)* 380' FROM THE TOP ELEV. OF DAM (ELEV. 41.3' MSL)⊛

ASSUME, EQUIV. LENGTH (HORIZ.) $\simeq 2/3 \times \frac{380}{4.2} H_D \simeq 60.3 H_D$ AND A DISCH. COEFF.:

$C \simeq 2.7$ $H_D = H - 5$

OR:

$$Q_B \simeq 163 (H-5)^{5/2}$$

iv) THEREFORE FOR ANY OVERTOPPING HEAD ($H \lesssim 15'$) ASSUME A DISCHARGE GIVEN APPROX. BY:

$$Q_0 = 870 H^{3/2} + 1590 (H-5)^{3/2} + 163 (H-5)^{5/2}$$

v) SURCHARGE HEIGHT TO PASS $Q_{P_1} = 48600$ CFS

$H_1 \simeq 9.6'$ (ABOVE SPILLW. CREST)

*FIELD OBSERVATION

⊛ NOTE. DRAWINGS SHOW ELEVATIONS IN NEW HAVEN DATUM (MEAN HIGH WATER) -
MSL (U.S.C.G.S. DATUM) ≅ NEW HAVEN DATUM (MHW) + 3.31'

HYDROLOGIC / HYDRAULIC INSPECTION

WHITNEY LAKE, HAMDEN, CT.

3-Cont'd) EFFECT OF SURCHARGE STORAGE

c) VOLUME OF SURCHARGE

ASSUME NORMAL POOL ELEV. 36.8' MSL (33.5 MHW) (i.e. 6" ABOVE SPILLW. CREST)

RESERVOIR AREA @ FLOW LINE = 178.3 AC. (NEW HAV. WATER CO. DATA, 1974)
(CF. AREA MEASURED 197 AC. ON USGS 1:24000 SCALE MAP)

∴ VOLUME OF SURCHARGE:

$$178.3 \times (9.6 - 0.5) \simeq 1620 \text{ AC-FT.}$$

$$D.A. = 37.4 \text{ SQ. MI.}$$

$$S_1 = \frac{1620}{37.4 \times 53.3} = 0.81''$$

d) PEAK OUTFLOW FOR STORAGE S_1

$$Q_{P_2} = Q_{P_1}\left(1 - \frac{S_1}{19}\right)$$

(SEE GUIDELINES - FROM ASSUMPT. OF TRIANGULAR HYDROGRAPH AND MPF RUNOFF IN NEW ENGLAND OF ±19")

$$\therefore Q_{P_2} = 48600\left(1 - \frac{0.81}{19}\right) = 48600 \times 0.96 = 46500 \text{ CFS}$$

(PRACTICALLY NO SURCHARGE STORAGE EFFECT)

FOR $Q_{P_2} = 46500$ CFS $H_2 \simeq 9.4'$ $S_2 = 0.80''$

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Project INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND Sheet 6 of 6
Computed By HLM Checked By D.SAHN Date 5/15/78
Field Book Ref. Other Refs. CE #27-531-GF Revisions

HYDROLOGIC / HYDRAULIC INSPECTION

WHITNEY LAKE, HAMDEN, CT.

3- Cont'd) EFFECT OF SURCHARGE STORAGE

e) RESULTING PEAK OUTFLOW:

$S_{AVE} = 0.805$

$$Q_{P_3} = 48600 \left(1 - \frac{0.805}{19}\right) \underset{\sim}{=} 46500^{CFS}$$

$\therefore \ H_3 \underset{\sim}{=} 9.4'$

f) SUMMARY

PEAK INFLOW: MPF $= Q_{P_1} = 48600^{CFS}$

PEAK OUTFLOW: $Q_{P_3} \underset{\sim}{=} 46500^{CFS}$

AVERAGE SURCHARGE: $H_3 \underset{\sim}{=} 9.4'$ OVER SPILLW. TO ±EL. 45.7' MSL

NOTE: DAM WILL BE OVERTOPPED (±) 4.4'

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Project INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND Sheet 1 of 2
Computed By HLL Checked By J. SHEN Date 5/9/78
Field Book Ref. Other Refs. CE #27-531-GF Revisions

HYDROLOGIC/HYDRAULIC INSPECTION

WHITNEY LAKE - SPILLWAY DATA AND DISCHARGE RATING

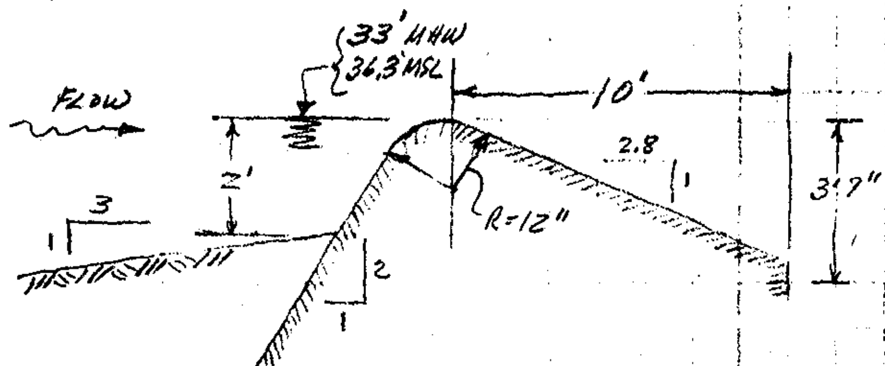
REF. NEW HAVEN WATER CO. - DWG. FILE #30059 - JAN 1917
"PLAN FOR RAISING AND LENGTHENING SPILLWAY" -
(ASSUMED TO REPRESENT "AS BUILT" CONDITIONS)

CREST ELEV. 36.3' MSL (33 MHW) TOP OF DAM EL. 41.3' MSL (38 MHW) (REF. WALL)

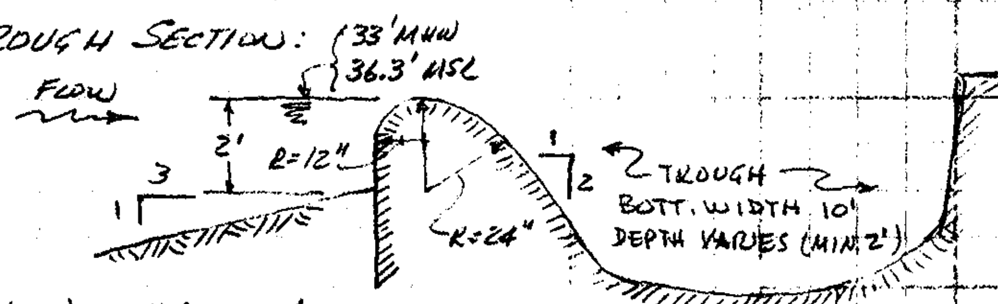
CREST LENGTH: MAIN SECTION = 182'
TROUGH SECTION = 68' (TO SIDE CHANNEL)
TOTAL LENGTH = 250'

SPILLWAY TYPICAL SECTIONS:

a) MAIN SECTION:



b) TROUGH SECTION:



NOTE: USCGS (MSL) ≡ MHW + 3.31'

HYDROLOGIC & HYDRAULIC INSPECTION - (SAMPLE COMPUTATION)

WHITNEY LAKE SPILLWAY DISCHARGE

1) SPILLWAY DISCHARGE COEFFICIENT.

WHITNEY LAKE HAS TWO TYPICAL SPILLWAY CROSS SECTIONS. - BOTH SECTIONS HAVE A CIRCULAR (COMPOUNDED) CREST AND SLOPED DOWNSTREAM FACE. THE MAIN SECTION HAS A 1 HOR. TO 2 VERT. SLOPED UPSTREAM FACE WHILE THE TROUGH SECTION HAS A VERTICAL UPSTREAM FACE. U/S DEPTH (P) IS IN BOTH SECTIONS P = 2'.

- AS AN APPROXIMATION, ASSUME THE FOLLOWING DISCHARGE COEFFICIENTS:

a) MAIN SECTION $C_1 = 3.5^{(1)}$ ∴ $C_1L_1 = 3.5 \times 182 = 637$

b) THROUGH SECTION $C_2 = 3.4^{(2)}$ $C_2L_2 = 3.4 \times 68 = 231$

$CL = 868$

∴ ASSUME FOR WHITNEY LAKE $Q = 868 H^{3/2}$ SAY, $Q = 870 H^{3/2}$

NOTES: THESE DISCHARGE COEFFICIENTS ARE ROUGH ESTIMATES. ACTUAL RATING CURVES AND DESIGN HEAD ARE NOT AVAILABLE.

(1) ROUNDED CREST $\overset{H}{3}:\overset{V}{1}$ & $\overset{H}{1}:\overset{V}{2}$ U/S HIGH HEAD SAY $\geq 5'$
$\sim \overset{H}{3}:\overset{V}{1}$ D/S

(2) OGEE TYPE (?) SIDE SPILLW. DISCH. - SIDE CHANNEL OF VARYING DEPTH. PROBABLE W/ SOME SUBMERGENCE AT HIGH FLOWS.

NOTE:

THESE COMPUTATIONS HAVE BEEN PERFORMED BASED UPON A DAM BREACH WITH A SURCHARGED WATER SURFACE ELEVATION. IN ACCORDANCE WITH NORMAL CORPS PROCEDURES, COMPUTATIONS ARE PERFORMED BASED UPON A WATER SURFACE ELEVATION AT THE TOP OF THE DAM. A DAM BREACH WITH THE WATER SURFACE AT THE TOP OF THE DAM AND WITHOUT HEAVY DOWNSTREAM CHANNEL FLOW COULD BE MORE CRITICAL THAN A DAM BREACH WITH A SURCHARGE. THE DIFFERENCE, IN THIS CASE, IS NOT SUBSTANTIAL.

APPENDIX

SECTION E: INVENTORY OF DAMS IN THE UNITED STATES

INVENTORY OF DAMS IN THE UNITED STATES

PAGE 16

10 MAR 78

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR. DIST.	STATE	COUNTY	CONGR. DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
CT	00119	NED	CT	009	05				WHITNEY LAKE DAM	1202.0	72546.0	12DEC73

POPULAR NAME	NAME OF IMPOUNDMENT

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST FROM DAM (MI.)	POPULATION
01	07	MILL RIVER	NEW HAVEN	1	135400

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT.)	HYDRAULIC HEIGHT (FT.)	IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT.)	IMPOUNDING CAPACITIES NORMAL (ACRE-FT.)
RECB	1861	S	22	20	2140	1926

REMARKS

D/S HAS	SPILLWAY CREST LENGTH	SPILLWAY TYPE	SPILLWAY WIDTH (FT.)	MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	POWER CAPACITY PROPOSED (MW)	NAVIGATION LOCKS NO.	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)
								0								

OWNER	ENGINEERING BY	CONSTRUCTION BY
NEW HAVEN WATER CO		

REGULATORY AGENCY DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
DEPT ENV PROT	04APR73	PA 571 SECT 25-11 ST OF CT

REMARKS